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DISCUSSION OF BUILDING FOUNDATIONS IN SAN FRANSICSO

WILLIAM W. MOORE,[1] M. ASCE and RICHARD S. HARDING,[2] J. M. ASCE.—The author presents a good description of the major types of subsurface conditions in San Fransicso. However, there are a few points concerning certain foundations referred to which might be further clarified.

In discussing the Pacific Gas and Electric Company Building, the author notes that the building was constructed in two stages—the original part was completed in 1925 and a new addition was erected in 1947. The presence of sandy "kidneys" offered a high frictional resistance during the driving of wood piling for the older building, and many of the piling had to be jetted to attain the desired penetration. A Vulcan No. 1 hammer (15,000 foot-pounds) was used to drive these piling. The author points out that the determination of these pile capacities by the Engineering News-Record formula indicates that the piling could safely support slightly less than three times the 54-kip design load. During our foundation investigation for the addition to the P. G. & E. Building in 1946, exploratory borings revealed the presence of 10 to 20 feet of sandy fill materials underlain by 30 to 45 feet of soft bay deposits. The fills offer appreciable resistance to the pile driving. However, with the passing of time, the weight of the fill causes the soft bay deposits to compress and subsidence occurs which results in "downdrag" on piling, thereby decreasing their permanent load-carrying capacity. This effect is illustrated on Figure 1, Computed Pile Capacities.* Beneath the soft bay deposits at the P. G. & E. Building site is a layer of firm sand ranging from 7 to 20 feet in thickness. This sand layer of varying thickness is probably the zone of sandy "kidneys" referred to in records of construction for the original portion of the building. This firm sand layer is in turn underlain by quite firm clayey soils. To gain the desired foundation support, it was necessary to depend on frictional resistance provided to piling by the firm sand layer and, in some areas, additional penetration into the still deeper firm clay was needed. Jetting through the deep sand would have reduced the frictional resistance provided by this material. By using a heavy hammer (a Vulcan 80-C furnishing 24,450 foot-pounds of impact) the piling for the new section of this building were successfully driven through the sand with jetting required in only a few instances.

A similar situation was encountered by us under part of the site for the Matson Building addition which adjoins the P. G. & E. Building on

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*These pile capacities were computed using the method described by William W. Moore, M. ASCE, in his paper "Experiences with Predetermining Pile Lengths".. Published in Transactions ASCE, Vol. 114, 1949, P. 351.

Market Street. This building was also constructed in two portions; the latter addition was erected in 1946. A test pile for the new addition was driven to practical refusal approximately 85 feet below the existing ground surface in the southwest portion of the property; in the northeast portion of the site another test pile refused at a depth of 58 feet below the surface. Exploratory borings drilled adjacent to these test piling showed that in the northeast portion of the site, the piling met refusal in a layer of dense sand immediately beneath the soft bay muds, and in the southwest portion refusal occurred in firm clays. Computed pile capacities indicated that the 25-ton design load and the eventual drag on the piling imposed by the subsidence of the soft bay deposits under the weight of overlying fills, would require the piling to be driven to 60 feet where the sand was present, and 85 feet elsewhere. Pile load tests carried out on a 55-foot pile driven some five feet into the sandy soils showed a capacity of 90 to 100 tons. The piling tested was driven to practical refusal with a Vulcan No. 1 hammer. When appropriate allowance was made for the potential drag due to subsidence of the fills, it was concluded that the pile tested would offer a safe permanent capacity of approximately 46 tons, providing a factor of safety of slightly less than 2 in relation to the permanent design loads. The Engineering News-Record formula would indicate that this pile could safely support at least 75 tons with a factor of safety of 6. In view of the conditions indicated, and the settlement behavior of both the P. G. & E. Building and the Matson Building, the writers believe that these are practical illustrations of conditions wherein the use of a dynamic formula for determining pile capacities would be not only erroneous, but actually dangerous.

The author has presented some interesting observed settlement data as related to the consolidation characteristics of San Francisco soils. Accurate evaluation of the settlement behavior of a building can be substantially improved by utilizing all available settlement records, in addition to a reasonable application of the theory of consolidation. This might be illustrated by our experience in estimating settlements for the addition to the Standard Oil Company Office Building in 1948. Consolidation tests performed on undisturbed core samples of the greenish-blue clay show results similar to those on the author's Diagram 5. The ultimate settlements which we computed for the older portion of this building in the course of our investigations for the planning of the new addition were about the same as were obtained by extrapolating the observed settlement records, which covered a period of 25 years. However, the originally computed theoretical time-rate of settlement was considerably slower than the observed time-rate. It is our belief that this difference resulted mainly from the presence of sand lenses of varying thicknesses and undetermined lateral extent, making the drainage structure of the deeper compressible clays quite indeterminate. Where these layers of sand are interconnected, the drainage for consolidation is materially hastened with a resulting increase in the settlement rate. A relatively small increase in the sand content of clayey soils can, also, provide this effect. A reasonable answer depends to a great degree on the care with which the field explorations are carried

out. The building site should be explored by enough test borings to provide a comprehensive picture of the subsurface conditions, and an experienced engineer should log the borings and obtain a sufficient number of undisturbed core samples to detect the presence of sandy gradations or lenses throughout the clayey strata. In evaluating the settlements expected for the Standard Oil Company Building addition, an empirical equation based on the observed settlement records for the adjacent older building was used to modify the theoretically computed time-rate of settlement. Where empirical data are not available in a case of this sort, engineering judgment and experience must be employed to evaluate the appropriate assumptions to be used in settlement analyses, and the range of accuracy which can be considered reliable.

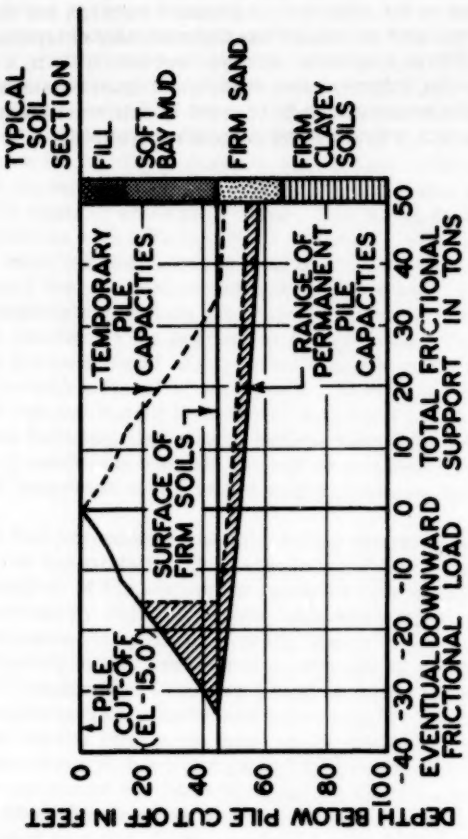

Fig. 1.—Computed Pile Capacities. 7-Inch Tip, Extension to General Office Building, Pacific Gas & Electric Company, San Francisco, California.

DISCUSSION OF TRAFFICABILITY OF SOIL AS RELATED TO MOBILITY OF VEHICHLES

HOWARD M. McMASTER,* A. M. ASCE.—The authors of this paper are to be commended for the realistic and comprehensive approach to the problem of trafficability as related to mobility of vehicles. The idea of establishing an analytical method of estimating performance of soils under traffic of all types of vehicles by means of one test is most worthwhile.

Examination of this paper rises a question on several factors concerning trafficability that might warrant a somewhat more detailed consideration. These factors are listed in order of decreasing importance as follows:

1. The change in shearing strength due to remoulding of the soil by a series of stress repetitions caused by a number of vehicle loads passing over the same area.
2. Effect of rate of loading on the shear strength of the soil.
3. Vegetation and its effect on trafficability.
4. The change in the shearing characteristics of a soil due to a change in moisture content.

In connection with the first factor, Messrs. Stewart and Weiss arrive at the conclusion that their approach to trafficability is applicable to trains of vehicles as well as individual vehicles. The necessity of soil trafficability evaluation for trains of vehicles is obvious when considering the military application. However, the authors have incorporated no recognition of the fact that when a series of vehicles pass over the same area of soil there can be a pronounced remoulding effect in the soil mass concerned. This may result in marked changes in shearing strength.

Examination of applicable publications indicates that the consistency, and therefore the shear strength of cohesive soils such as clay, is changed by the remoulding of the soil[1].

The term sensitivity is used to describe the susceptibility of a soil to a consistency change through remoulding. It is stated that the sensitivity of a given clay can vary with its moisture content. There is also a consistency variation between different clays.[1]

The degree of sensitivity is defined as the ratio of the Unconfined Compressive Strength Undisturbed to the Unconfined Compressive Strength Remoulded.[1,3] The values of sensitivity range from 2 to 4 for most clays, and from 4 to 8 or more for sensitive or extrasensitive clays.[1] All clays are originally weakened by remoulding.[3] The fact that some remoulded clays regain their strength after a period of time[5] would not be applicable to the study of trafficability in view of the short time intervals involved.

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Examination of sand indicates that the effect of remoulding is not significant unless it causes a change in the density. If this change occurs, some idea of the induced error may be obtained from the results of the slower shear tests.[1] These tests indicated a possible variation of 27 to 30 degrees for the loose state to 30 to 35 degrees when the soil is in the dense state.

It is apparent from these findings that as more and more vehicles pass over the same area, the soil mass involved will become gradually remoulded until a later vehicle in the train may bog down completely. Therefore, an evaluation of the remoulding effect is essential if the proposed approach to trafficability is to be utilized with a maximum of accuracy where trains of vehicles are to be involved.

The investigation on the strength of soils under dynamic loads conducted by A. Casagrande and W. L. Shannon[3] shows that for undisturbed, soft, organic clay, the strength in the triaxial compression test for the fastest rate of loading or transient loading, was 40 per cent greater than for the slowest rate of loading. In general it was found that for the weaker clays the fastest rate of loading strength is about twice the 10 minute strength and for the stronger clays the fastest rate of loading strength is about 50 per cent greater than the 10 minute loading strength. In the latter, the transient loading time was about 0.02 sec. Tests were run on undisturbed samples in both the triaxial compression test and the unconfined compression test.

Results from specimens of clay remoulded at the liquid limit indicate a 25 per cent greater strength for the faster loading.[4]

Tests similar to those on clays were run on sands with the result that the faster rate of loading provided a 10 per cent greater strength than the 10 minute loading strength.

In order to properly evaluate the trafficability of soils and the effect of the rate of loading factor it would seem necessary to establish some relationship between the rate of loading for the proposed soil truss and the rate of loading that might be expected under the track of the vehicle in the field. Possible values of loading rate might be 2 seconds for the soil truss and 0.10 seconds to 0.02 seconds when load is applied by vehicle. If some correlation is possible, the results of the in-place soil truss test could be accepted with more confidence.

A somewhat less important but nonetheless significant factor is the effect of vegetation on soil shear strength.

If cohesive soils are considered, there is no direct effect on the shear characteristics. However, evaluation of the granular soils such as a sand reveals the fact that in many cases the presence of vegetation and the associated root structure with its confining action is the only factor allowing the passage of traffic.

The moisture content of a soil can affect its shear strength.[1] When considering cohesive types of soils, the moisture content of a tested soil mass becomes important only when it changes before or during the passage of the traffic. Because of this the authors should evaluate the possible change in shear strength due to changes in moisture content.

The moisture content of granular soils is of lesser importance and need not be considered here.

The paper indicates that only a comparatively limited number of observations were made. It is felt that a much more comprehensive set of data should be collected before general conclusions are drawn regarding the conformance of field test results with theory.

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3. A. Casagrande and W. L. Shannon; Strength of Soils under Dynamic Loads; Paper No. 2378 Transactions ASCS Vol. 114 1949.
4. Taylor, D. W.; Progress Report on Triaxial Shear Research and Pressure Distribution Studies on Soils, U. S. Waterways Experiment Station, Vicksburg, Miss., April 1947, p. 95.
5. Tschebotarioff, Gregory P.; Soil Mechanics, Foundations, and Earth Structures; McGraw-Hill Book Co., Inc., 1951.

DISCUSSION OF STRENGTH CHARACTERISTICS OF COMPACTED CLAYS

H. J. GIBBS[1] A.M. ASCE and J. W. HILF[1] A.M. ASCE.—The author has presented a relationship between volume change and strength of unsaturated remolded clays which is of value in analyzing triaxial shear test results, but which the writers believe is not a satisfactory substitute for the Mohr strength envelope. Also, the implications in the author's paper that it may be desirable to reduce the compactive effort on clay soils in earth dam embankments in order to obtain greater strength are not substantiated by Bureau tests nor by theory.

The plot of void ratio at failure, for a given initial void ratio, as ordinate; against logarithm of maximum deviator stress, as abscissa is essentially a shearing strength—volume strain diagram which shows the important role played by volume change during shear. Volume decrease in soil, during shear or otherwise, depends on effective normal compressive stresses which, in turn, are functions of the applied stresses, of the compressibility of the pore fluid, and of the amount of drainage permitted. For the past 15 years the Bureau of Reclamation has been using the relation between shearing strength and effective stress in interpreting triaxial shear tests on unsaturated impervious soils and applying them in the design of earth dams and canal embankments. A Mohr strength envelope is obtained from sealed tests in which effective normal stresses are determined by subtracting measured pore pressures from the applied normal stresses. In order to compare the two types of plots, the author's and the Bureau's, the following data from tests reported in 1950[2] have been analyzed. The particular test series were selected because they also contain results pertinent to the question of compaction versus consolidation.

The soil used in these tests was a lean clay from the vicinity of Vallecito Dam, Colorado. It had the following properties:

Specific gravity of grains	2.739
Liquid limit	26.6 percent
Plasticity index	7.3 percent
Gradation; maximum size	1/4 inch
27 percent larger than No. 200 sieve	
23 percent smaller than 0.005 mm	
Proctor compaction (USBR standard)	

1. Engineers, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.
2. "Shear Characteristics of Remolded Earth Materials," by A. A. Wagner, Special Publication No. 106, ASTM (1950), pp. 192-227.

Maximum dry density	116.0 pcf
Optimum moisture content	13.4 percent
Permeability at Proctor maximum dry density	0.06 foot per year

Figure 1 gives the triaxial shear results for this soil tested under six different conditions as follows:

Tests at the same initial density

No. 18T—e_0 = 0.525 ± 0.001 W = 9.67 ± 0.02 %
No. 17T—e_0 = 0.525 ± 0.003 W = 11.39 ± 0.04 %
No. 16T—e_0 = 0.519 ± 0.001 W = 13.55 ± 0.03 %
No. 15T—e_0 = 0.525 ± 0.003 W = 15.21 ± 0.02 %

Tests at different density but at moisture content same as No. 16T

No. 14T—e_0 = 0.499 ± 0.004 W = 13.50 ± 0.11 %

Tests at different density but at moisture content same as No. 15T

No. 13T—e_0 = 0.500 ± 0.003 W = 15.25 ± 0.07 %

Figure 2 is a combined plot of the Mohr envelopes shown at the bottom of Figure 1. All of these tests were made under sealed conditions; therefore, the moisture content of each test remained constant during the test. The shear specimens were 3-1/4 inches in diameter by 9 inches long, and were compacted by impact.

For constant placement density, the results show that the condition of lowest moisture contant (Test 18T) had the highest cohesion and that the cohesion consistently became smaller with increase in initial moisture content (Tests 17T, 16T, and 15T, respectively). The angle of internal friction was less affected by moisture content, but, with the exception of Test 17T, it also became smaller as initial moisture contant increased. These results are related to the initial density, to the volume change which occurs during the test, and to the density at failure. In Figure 3, the compressive strength (deviator stress)-void ratio at failure curves have relative positions which reflect these shear characteristics. Test 18T has a curve at the highest position, and Tests 17T, 16T, and 15T have lower positions in the order of increasing initial moisture content. A similar comparison can be made for tests at different initial densities but at the same moisture content. For this comparison, cohesion is relatively little affected by increased initial density. However, the tests at higher initial density have higher angles of internal friction, as shown in Figure 2, by comparing Test 14T to 16T and Test 13T to 15T. The same relationship is also reflected in Figure 3.

The failure criterion used for the data shown in Figures 1, 2, and 3 was the maximum principal stress ratio.[3] In order to make a direct

3. "The Use of the Maximum Principal Stress Ratio as the Failure Criterion in Evaluating Triaxial Shear Tests on Earth Materials," by W. G. Holtz, Proceedings, ASTM, Vol. 47, 1947, p. 1067.

comparison with the author's data, the results of Tests 13T, 14T, 15T, and 16T have been replotted in Figure 4, using the original test data and the maximum deviator stress failure criterion. The relative positions of the test curves in Figures 3 and 4 are approximately the same which indicates that the effect of using different failure criteria was small. These tests on the Vallecito soil do not substantiate the author's Conclusion 3, since the strength for Test 13T (the one with the greater density) was greater than for 15T, and the strength for Test 14T (the one with the greater density) was greater than for 16T for most of the range of e_f.

In his discussion of results, under "Compaction vs Consolidation," the author states that the reasons for the apparent superiority of the consolidation process over the compaction process, as found from his test results, is not immediately evident. In view of the reverse relationship found in the foregoing Bureau tests, a possible theoretical explanation for the conflicting data is offered. For a given water content, a cohesive soil can be compacted to a low void ratio which results in very little air volume. When the soil is loaded by superimposed fill, the small air content and the virtual imperviousness of the soil preclude (through the mechanism of pore pressure) the development of effective stress and frictional shearing resistance, without affecting the shearing stresses induced by the loading. There are two apparent methods of correcting this condition in impervious soils: (1) to lessen the compactive effort and obtain larger quantities of air in the soil prior to loading, and (2) to control the moisture content so that an appreciable quantity of air remains in the soil after it has been compacted to a high density. The Buearu of Reclamation has used the latter method successfully. The fallacy of using Method (1) is that compressibility of the soil (the volume change-effective stress relation) is greater for initially low density soils than for initially high density soils, hence pore pressure development may be facilitated despite the increase in air volume. The relation of air content and compressibility to pore pressure is given by the following equation[4] for unsaturated soil for the condition of no drainage:

$$u = \frac{u_a \Delta}{V_a + hV_w - \Delta}, \quad \Delta \leqq V_a \qquad \text{Equation (1)}$$

where

u_a = initial pore pressure (usually considered as atmospheric pressure)
V_w = volume of water in the soil in percent of initial soil volume
h = capacity of water to dissolve air by volume, from Henry's law (0.02 at 20° C)
V_a = initial volume of free air in the soil mass in percent of total initial volume
Δ = volume change in percent of total initial soil volume (compression is considered positive)
u = developed pore pressure (gage pressure)

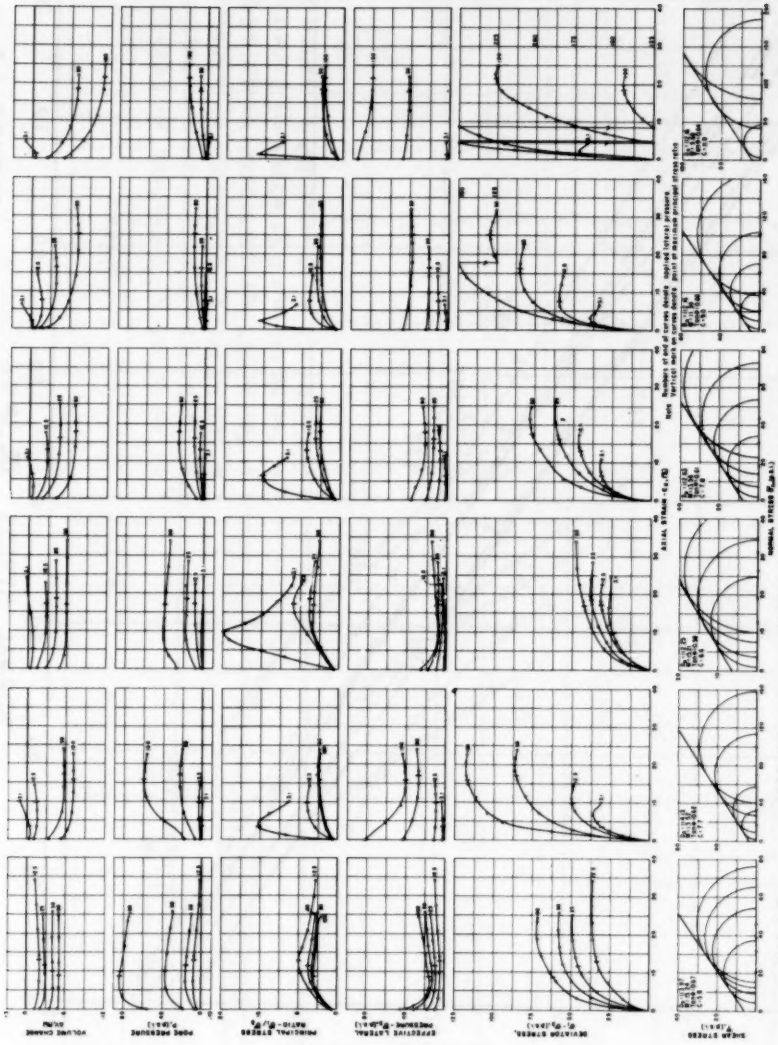
4. "Estimating Construction Pore Pressures in Rolled Earth Dams," by J. W. Hilf, Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 3, 1948, p

Equation (1) shows that the pore pressure of an impervious soil at a given water content increases as compressibility increases and as the air content decreases. There is a possibility that decrease in density of some soils at a given water content may result in sufficient air to more-than-offset their increased compressibility, which would account for strength results similar to those obtained by the author. In most soils used for dam embankments, the increase in compressibility would control; pore pressures would increase and strength would decrease, which would account for the Vallecito test results. However, for the virtually sealed condition which exists in the impervious core of a rolled earth dam during construction, the pore pressure formula strongly indicates the desirability of controlling the water content and of using heavy compaction to obtain both appreciable air content and low volume compressibility of the placed soil. The treatment of expansive clays in the subgrades of lightly loaded structures must be clearly distinguished from the placing of nonexpansive clays in earth dams where the confining stresses are high. The possibility of reducing compaction in the former case has been considered by the Bureau to reduce swell under canal linings.[5]

The method of triaxial shear testing and interpreting test results described in Mr. Wagner's paper[2] is based on effective stresses; pore pressures are measured, and a true angle of internal friction is determined from the Mohr strength envelope. This requires estimating pore pressures in the prototype in order to calculate the stability of rolled earth dams. The pore pressures are obtained from the flow net for the steady-stage seepage condition; and are estimated by the use of Boyle's and Henry's laws and consolidometer tests for the construction condition.[4] Although it is based on assumptions which only approximate field conditions, this procedure is both simple and practicable. Improvements in testing techniques and in analysis are in progress. On the other hand, practical application of the author's proposal stated in his Conclusion 4 appears to be a large order for theoretical soil mechanics. Prediction of the void ratio at failure in an unsaturated clay embankment from given placement conditions would appear to require a solution of the stress distribution in the embankment as well as a prediction of the variation of pore pressures with time and space.

Despite the relation shown by the author, satisfactory interpretation and application of triaxial shear test results on unsaturated cohesive soils cannot be made without considering the stresses in the pore fluid of the soil mass. The effective Mohr strength envelope and its inclination, the true angle of internal friction, satisfactorily explain the mechanism of shear failure in all soils. They can be obtained by proper test procedure and are relatively easy to apply in design. While the author's plot of void ratio at failure against shearing strength can be used in conjunction with the Mohr envelope as an additional check on test results, it is not a satisfactory substitute for the envelope.

5. "Engineering Properties of Expansive Clays," by W. G. Holtz and H. J. Gibbs, presented at the Centennial Meeting of the American Society of Civil Engineers, September 1952.



TRIAXIAL SHEAR TEST-"TYPICAL MATERIALS"-SAMPLE NO. 007-B (Clay loam)

FIGURE 1

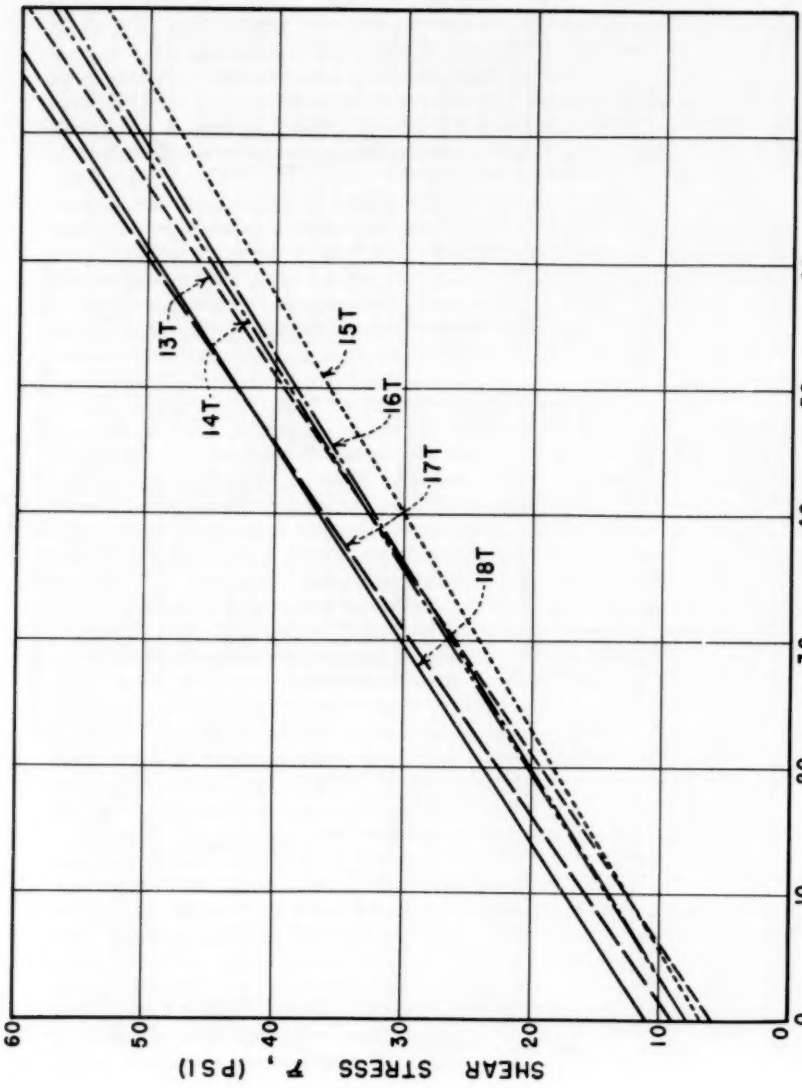

SHEAR STRESS τ, (PSI)
NORMAL STRESS σn, (PSI)
0
10
20
30
40
50
60
70
80
13T
14T
15T
16T
17T
18T

FIGURE 2

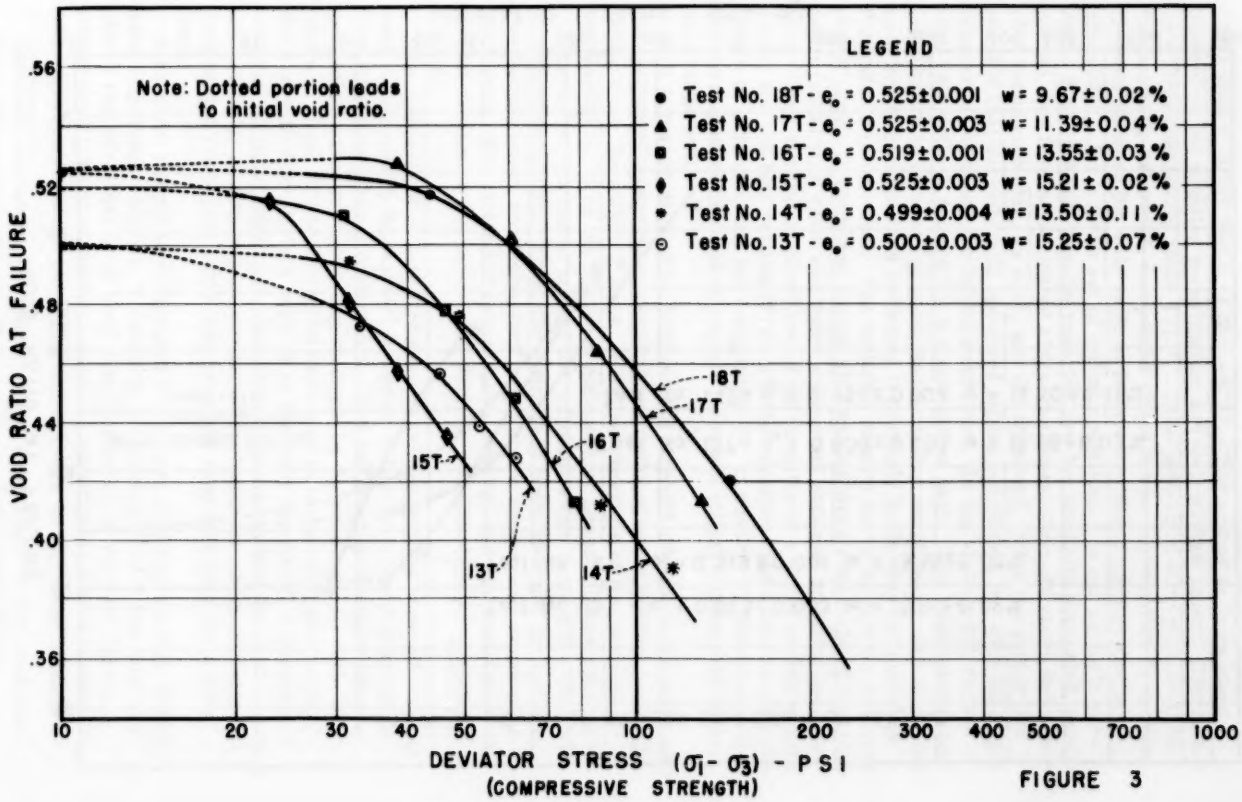

LEGEND
Test No. 18T - e_o = 0.525±0.001 w = 9.67±0.02 %
Test No. 17T - e_o = 0.525±0.003 w = 11.39±0.04 %
Test No. 16T - e_o = 0.519±0.001 w = 13.55±0.03 %
Test No. 15T - e_o = 0.525±0.003 w = 15.21±0.02 %
Test No. 14T - e_o = 0.499±0.004 w = 13.50±0.11 %
Test No. 13T - e_o = 0.500±0.003 w = 15.25±0.07 %
Note: Dotted portion leads to initial void ratio.
18T
17T
16T
15T
14T
13T
VOID RATIO AT FAILURE
.56
.52
.48
.44
.40
.36
10
20
30
40
50
70
100
200
300
400
500
700
1000
DEVIATOR STRESS $(\sigma_1 - \sigma_3)$ - P S I
(COMPRESSIVE STRENGTH)

FIGURE 3

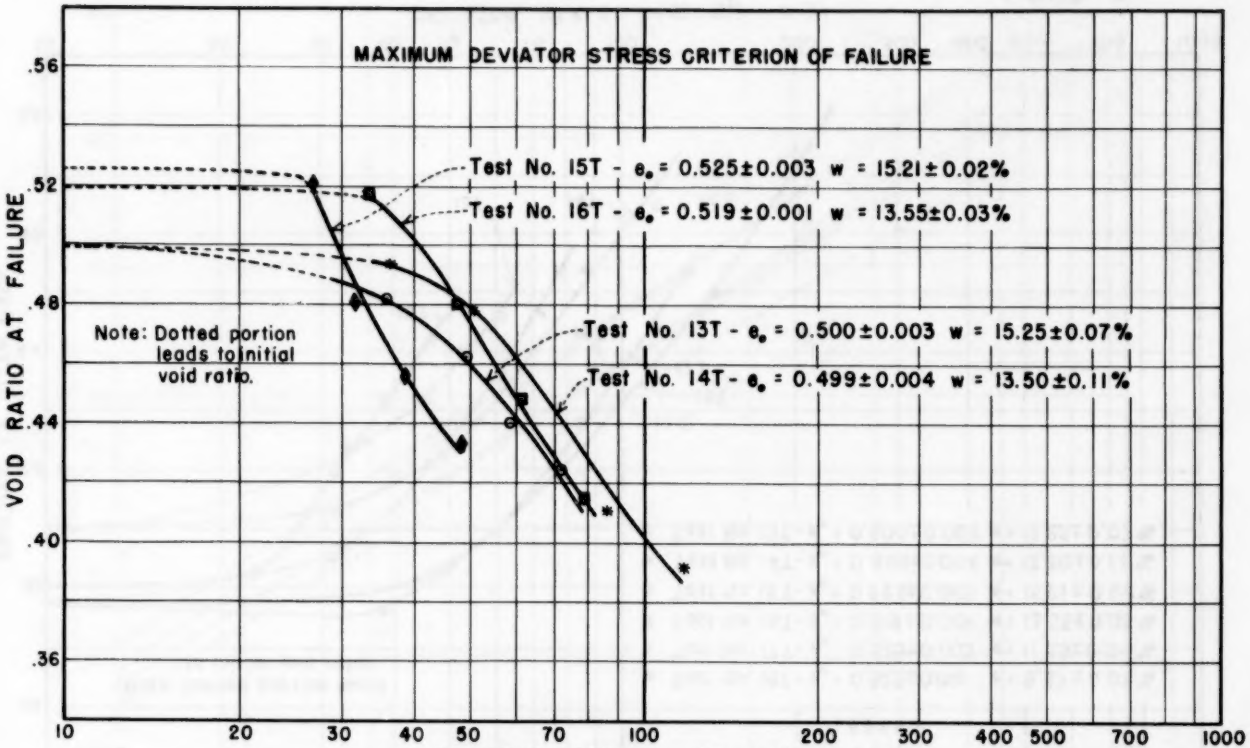

MAXIMUM DEVIATOR STRESS CRITERION OF FAILURE
Test No. 15T - e_o = 0.525±0.003 w = 15.21±0.02%
Test No. 16T - e_o = 0.519±0.001 w = 13.55±0.03%
Test No. 13T - e_o = 0.500±0.003 w = 15.25±0.07%
Test No. 14T - e_o = 0.499±0.004 w = 13.50±0.11%
Note: Dotted portion leads to initial void ratio.
VOID RATIO AT FAILURE
.56
.52
.48
.44
.40
.36
10
20
30
40
50
70
100
200
300
400
500
700
1000
DEVIATOR STRESS $(\sigma_1 - \sigma_3)$ - PSI
(COMPRESSIVE STRENGTH)

FIGURE 4

JAMES K. MITCHELL, J.M., ASCE[1],—The author is to be complimented on his detailed investigation and well written paper. The author's first conclusion (page 15), ".....the pattern of strength variation for partially saturated, compacted clays can be represented by a relationship between the void ratio at failure and the.....compressive strength.....for a given set of initial conditions," is not entirely correct. Since the test results reported are for partially saturated, compacted, and then consolidated clays, the actual strength characteristics of a compacted clay, as such, are not apparent. What the author has found is that not only with specified initial conditions, but also specified manner of obtaining the initial conditions, the log of strength varies with void ratio at failure for a consolidated soil. This relationship has been known to be true for saturated soils for many years, and more recent investigations have indicated it probably holds true for partially saturated soils as well. The author's results serve as further evidence that it does. In each test series all samples were consolidated from the same initial void ratio and water content as obtained by static compaction. Thus, the initial structure of all samples in a given series was the same, and the test results (Figs. 12-20) represent the effect of consolidation on strength, and not compaction as, in reality, all the compaction served to accomplish was to impart a certain initial undisturbed structure to the soil.

The actual relationships between the strength of compacted soils and the method of compaction, density, and molding water content cannot be drawn from the author's tests. With the exception of the samples in test series No. 5 on the Modified Mississippi Loess, all samples were compacted with a static load at the optimum water content for standard Proctor compaction. Thus, the location of these samples on a static compaction curve corresponding to the static compactive effort used is not known. The method of compaction, density, and molding water content all determine the particle to particle relationships, or structure, of the soil.

The importance of soil structure in influencing the strength has not been mentioned by the author; however, it has been found of primary importance by other investigators. The superiority of consolidation over compaction in increasing strength by decreasing void ratio as shown in Figs. 15 and 20 can be explained in terms of soil structure. More specifically, in this case, it is probably the effect of duration of load that is most important; in a consolidating system the load is acting continuously, and there is a greater opportunity for close particle approach at a given void ratio than when the load acts only for a short time.

Structural effects in compacted soils were noted by the U.S. Waterways Experiment Station, Vicksburg, Mississippi[1, 2]; and they concluded that structure is of such significance as to warrant listing it among the primary variables. Tsien[3] investigated this structure in detail and found, as did the WES, that not only do dry-side* samples

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*compacted at a water contant less than optimum

have a high strength explainable only in terms of an "additional structural strength," but also, if a dry-side sample is soaked to a water content equal to that of a wet-side** sample at the same void ratio, it still has a higher strength than the wet-side sample. Tsien found that a dynamically compacted, soaked sample had approximately 1.25 times the strength of a statically compacted, soaked sample at the same void ratio; Eustis and McRae[2] found the strength of dynamically compacted, soaked samples to be 2.5 times that of statically compacted, soaked samples. Thus, there can be little question that the method of compaction exerts an influence on strength. This point did not show up in the author's investigation because all samples were compacted statically at a given water content; however, it is felt that a description of the compaction method should be included with the "given initial conditions" when curves of the nature of Figs. 12-20 are considered.

Data showing the effect of structure on strength have been obtained by the writer and are given in Table 2 and Figs. 1, 2, and 3. For these tests a sandy clay from Fort Belvoir, Va. was used; properties of this soil (based on -#10 sieve fraction) are listed in Table 1. Samples were compacted by means of the Harvard Miniature Compaction Apparatus developed by Wilson[4] (kneading compaction), extruded, and tested immediately in unconfined compression. Fig. 1 presents density vs. molding water content. Fig. 2, (compressive strength vs. molding water content) shows a decreasing strength with increasing water content. Although no good correlation exists between axial strain at failure with either water content or strength, the data in Table 2 indicates a tendency towards increasing axial strain at failure with increasing water content and decreasing strength.

In Fig. 3, where initial void ratio is plotted against the log of unconfined compressive strength, a very interesting relationship may be noted. (Values determined from samples compacted at or wetter than optimum are represented by triangles and values for samples compacted dry of optimum, by circles.) Fig. 3 shows a linear relationship between the initial void ratio and the log of unconfined compressive strength for samples molded wet of optimum water content. No such relationship exists for those samples molded dry of optimum water content. The scatter of dry-side points is probably the result of minor variations in compaction procedure as dry-side samples have been noted in these and other tests to be very sensitive to procedure when using the Harvard Miniature Compaction Apparatus. Even so, there is little reason to expect that there should be a direct correlation between void ratio or water content and strength for dry-side samples because structure cannot, at the moment, be expressed by any single parameter. Concepts suggested by Lambe[5] and current investigations at M.I.T. are leading to the conclusion that structure in a given soil is a function of capillary tensions, water adsorption characteristics, nature of adsorbed cations, particle orientation, and interparticle spacing. Most likely, at wetter than optimum water contents the larger amounts of water present lead to appreciable pore pressures during compaction and a marked reduction in capillary tensions with the result that orientation tends to become somewhat

** compacted at a water content greater than optimum

constant and particle spacing, which can be conveniently expressed by void ratio, becomes the controlling variable.

In all probability, had the data in Fig. 3 been plotted against void ratio at failure the scatter would have been reduced. It has been long recognized that failure void ratio has much more theoretical justification than initial void ratio in all strength studies; however, failure void ratio is a very difficult property to measure. Consolidation pressure or initial void ratio must be used in most investigations. Although the procedure developed by the author for a volume measurement at failure (described on page 11), was satisfactory for his tests, such a procedure would not be possible for the dry-side samples reported in Fig. 3 where failure was accompanied by splitting and crumbling of the sample. Initial and final void ratios were probably about equal for the dry-side samples as strain at failure was small.

The results in Fig. 3 are in agreement with Tsien's findings and show that there is an additional structural strength present in dry-side samples that cannot be explained on the basis of initial or final void ratio, water content, or degree of saturation. At the present time only general, qualitative explanations can be given for this phenomenon but it is hoped that, as more information on the fundamental nature of soil becomes available, quantitative explanations will be possible.

The following suggestions are made:

1. The author's paper be viewed as the strength properties of compacted and then consolidated soil, with conditions prior to consolidation strictly defined.
2. The effect of structure on the strength of compacted soils not be neglected. This structure, which is a function of soil composition, adsorbed water characteristics, adsorbed cation types, water content, particle orientation, and particle spacing, is much more important in dry-side samples than in wet-side samples.
3. A more accessible property than failure void ratio be sought for the purpose of correlating strength results.

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2. Eustis, J. B. and McRae, J. L., 1945, "Stress-Strain Characteristics of Compacted Soil Systems," Presented at Annual Meeting ASCE, New York.
3. Tsien, Shou-I, 1946, "A Study of Structure in Compacted Soils," ScD. Thesis, M.I.T., Cambridge, Mass.
4. Wilson, S. D., 1950, "Small Soil Compaction Apparatus Duplicates Field Results Closely," Eng. News Record, Nov. 2, 1950.
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TABLE 1

CLASSIFICATION AND MINERALOGICAL DATA ON FORT BELVOIR SANDY CLAY

Specific gravity of solids	2.72	
Grain size distribution		
D_{60} (mm)	0.20	
D_{10} (mm)	0.02	
D_{60}/D_{10}	10	
Harvard Miniature Compaction		
Optimum moisture content (%)	19.0	
Maximum dry unit wt. (lb/ft^3)	106	
Atterberg limits		
Liquid limit (%)	45	
Plastic limit (%)	21	
Plasticity index (%)	24	
Mineralogical composition (-140 sieve fraction = 50% total)		
Kaolinite)	35%)	) Random stratification
Hydrous mica)	5%)	
Montmorillonite	9%	
Quartz	40%	
Free iron oxide	3%	

TABLE 2

SUMMARY OF TEST RESULTS

Test No.	Molding Water %	Dry Density lb/cu ft	Void Ratio Initial	Saturation %	Axial Strain %	Compressive Strength $(\sigma_1-\sigma_3)$ psi
1	15.1	98.7	0.721	59.1	2.0	34
2	15.8	97.2	0.750	57.4	1.4	30
3	16.0	106.4	0.599	72.6	4.1	57
4	16.1	100.1	0.700	62.6	2.3	47
5	16.3	106.1	0.604	73.5	4.6	55
6	16.4	106.1	0.603	74.0	3.9	47
7	16.7	98.6	0.721	63.0	1.8	27
8	16.7	101.6	0.671	67.6	2.8	41
9	17.0	102.0	0.667	69.3	3.0	46
10	17.0	106.0	0.603	76.6	3.9	41
11	17.1	100.5	0.690	67.5	2.8	34
12	17.1	105.5	0.610	76.1	3.6	45
13	17.3	103.7	0.640	73.0	4.3	48
14	17.4	98.4	0.730	64.9	2.7	36
15	17.6	105.1	0.615	77.7	4.8	28
16	17.8	107.4	0.583	83.5	6.0	48
17	17.8	102.2	0.662	72.1	3.0	36
18	18.4	105.9	0.606	82.5	4.3	23
19	18.4	106.1	0.599	83.4	4.1	31
20	18.9	104.9	0.620	82.9	3.2	35
21	18.9	102.9	0.654	78.5	4.3	22
22	19.0	108.0	0.575	90.0	7.8	34
23	19.0	106.3	0.600	86.4	6.4	28
24	19.1	107.0	0.587	88.3	8.9	26
25	19.3	106.9	0.590	89.0	5.7	32
26	19.3	104.3	0.629	83.5	5.0	25
27	19.3	105.5	0.611	86.0	5.7	23
28	19.4	106.7	0.595	89.0	6.0	33

TABLE 2 (cont'd)

SUMMARY OF TEST RESULTS

Test No.	Molding Water %	Dry Density lb/cu ft	Void Ratio Initial	Saturation %	Axial Strain %	Compressive Strength $(\sigma_1-\sigma_3)$ psi
29	19.5	104.9	0.620	85.3	8.5	27
30	19.7	103.5	0.640	83.6	12.8	24
31	19.8	106.6	0.595	90.5	7.8	31
32	20.0	105.6	0.611	89.0	7.1	30
33	20.0	104.6	0.629	86.8	4.3	32
34	20.3	105.0	0.620	89.1	10.7	25
35	20.9	103.5	0.640	88.5	10.7	25
36	21.2	102.2	0.663	87.2	10.3	22
37	21.2	102.7	0.658	88.0	10.7	20
38	21.7	102.8	0.654	90.3	10.0	22
39	21.7	101.7	0.671	88.0	12.8	17
40	22.1	98.7	0.721	83.1	7.5	13
41	22.5	100.0	0.703	87.3	12.1	14
42	22.5	101.0	0.685	89.5	12.8	18
43	22.7	100.7	0.690	89.9	9.2	16
44	22.9	98.2	0.732	85.4	11.4	13
45	24.6	96.5	0.761	87.9	10.7	11
46	24.7	98.5	0.727	92.5	12.8	14
47	25.4	98.0	0.737	94.0	12.8	13

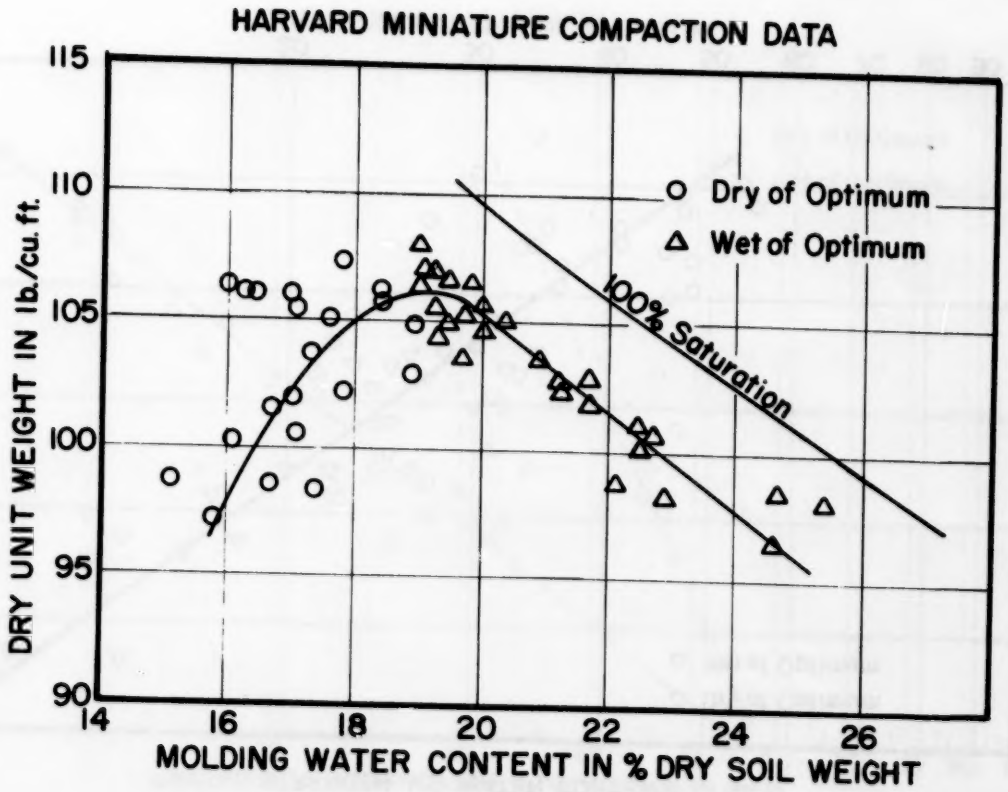

HARVARD MINIATURE COMPACTION DATA
DRY UNIT WEIGHT IN lb./cu. ft.
115
110
105
100
95
90
14
16
18
20
22
24
26
MOLDING WATER CONTENT IN % DRY SOIL WEIGHT
Dry of Optimum
Wet of Optimum
100% Saturation

Fig. 1

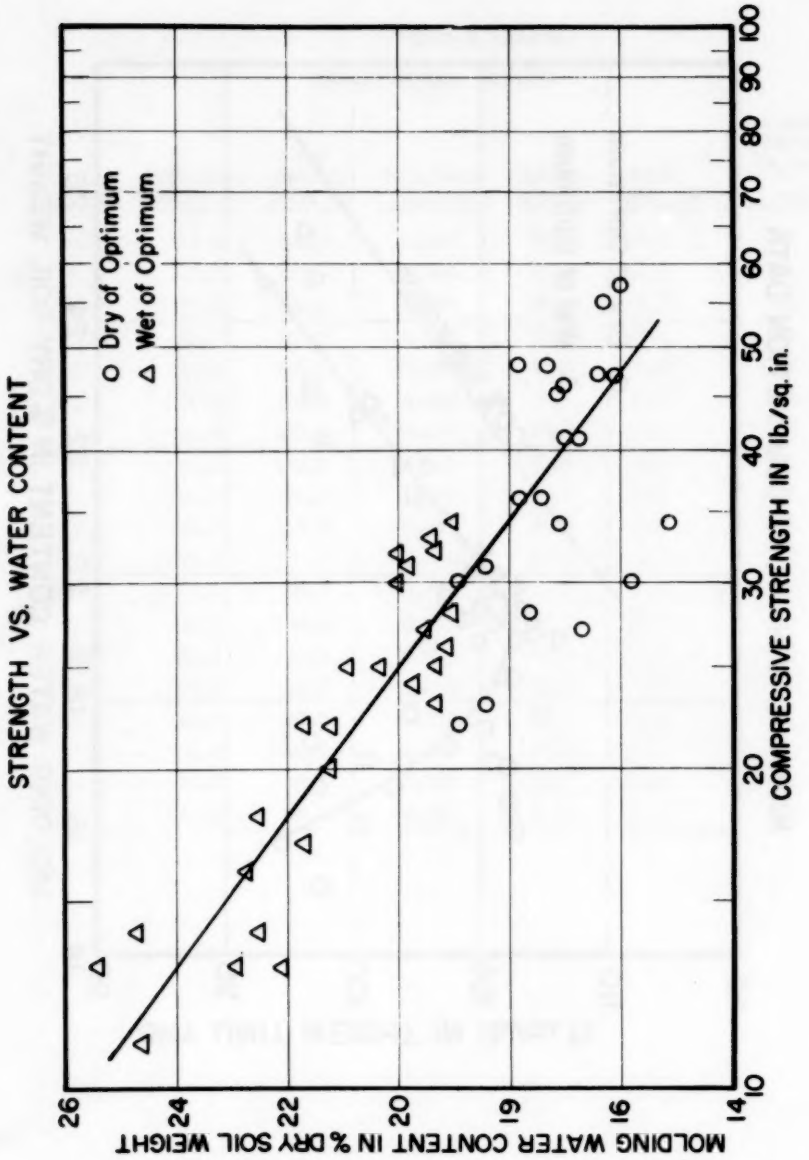
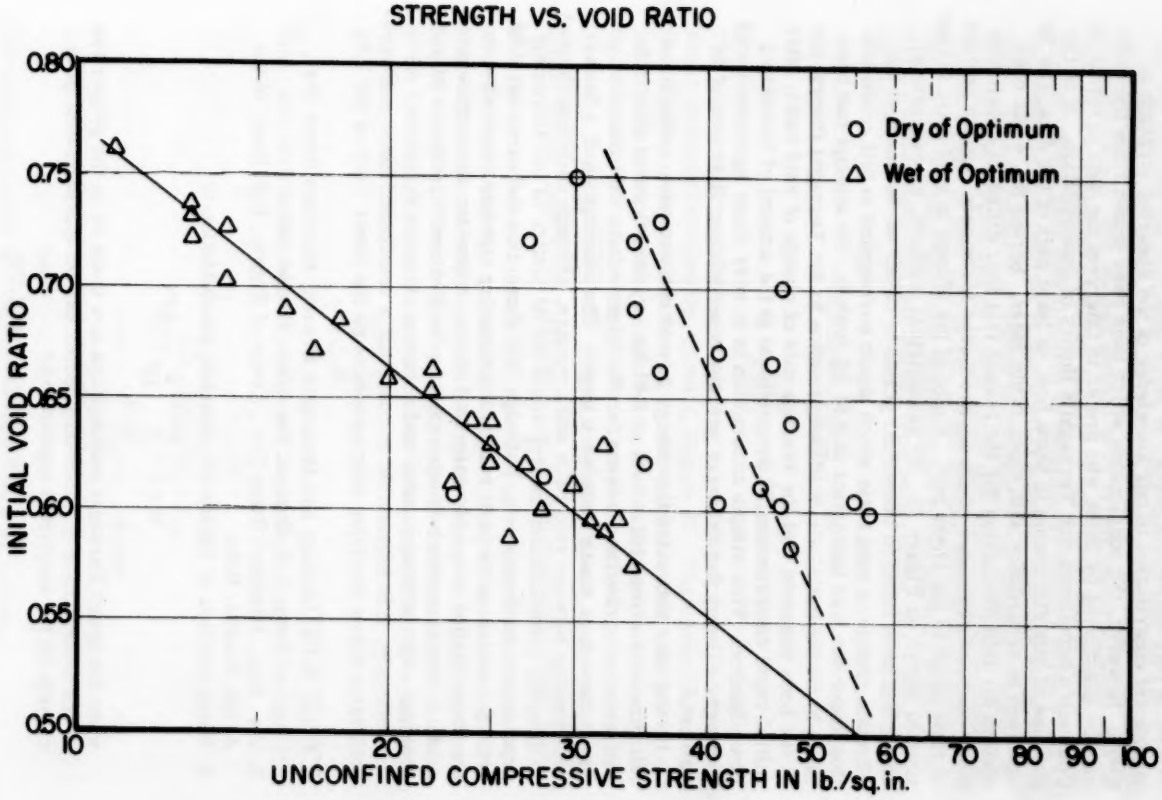

STRENGTH VS. WATER CONTENT
Dry of Optimum
Wet of Optimum
COMPRESSIVE STRENGTH IN lb./sq. in.
10
20
30
40
50
60
70
80
90
100
MOLDING WATER CONTENT IN % DRY SOIL WEIGHT
26
24
22
20
19
16
14

Fig. 2


STRENGTH VS. VOID RATIO
INITIAL VOID RATIO
0.80
0.75
0.70
0.65
0.60
0.55
0.50
10
20
30
40
50
60
70
80
90
100
UNCONFINED COMPRESSIVE STRENGTH IN lb./sq. in.
Dry of Optimum
Wet of Optimum

518-25

Fig. 3

PRESTON T. BENNETT,[1] M. ASCE and PAUL E. WOHLT,[2] A.M. ASCE.—Everyone should agree that the author's test data constitute a significant contribution to our knowledge of the shearing strength of clays. However, the conclusions which have been drawn from the tests may be questioned by those who prefer to interpret the data in terms of Coulomb's hypothesis and Terzaghi's theory of consolidation. It is the purpose of this discussion to show how, at least part of the data, can be analysed in accordance with those basic ideas. Series No. 4 has been chosen for this discussion for the reason that four different specimens were consolidated under confining pressures for widely varying periods of time prior to the shear test. A plot of the change in void ratio versus time is shown on Figure 25. By geometrical methods, an exponential curve has been fitted to these four points to obtain an estimate of the ultimate change in void ratio which would correspond to full consolidation under the load increment of 2.81 kg./sq.cm. No attempt has been made to fit the initial air-void compression to the Terzaghi theory, but it has been assumed that the average rate of change of void ratio, after initial rapid compression, is proportional to the amount of remaining consolidation. This simple assumption is in very close agreement with Terzaghi's theory for values of average consolidation in excess of 20 percent.[3]

Having thus estimated the change in void ratio corresponding to an intergranular stress increment of 2.81 kg./sq.cm., the next step is to estimate intergranular stresses for the intermediate void ratios at which these four shear tests were begun. The assumption of a linear relationship between void ratio and pressure, although customarily used to simplify consolidation theory, would not do justice to the extremely precise test methods used. Rutledge (25) shows that the curve for void ratio (or moisture content) versus log shearing strength is parallel to the consolidation curve for saturated clays. A similar assumption was made in this case which is equivalent to the Coulomb hypothesis of a constant ratio between normal and shearing stresses at failure.

Obviously, it is impossible to construct a semilogarithmic pressure-void ratio curve involving zero pressure as the lower limit of the 2.81

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3. Using notation of Taylor (3), average consolidation is

$$U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \epsilon^{-M^2 T}$$

when the usual Terzaghi assumptions are made as to the properties of the materials. If $T > 0.06$ this series can be approximated quite closely by the empirical expression

$$U = 0.17 + 0.83 (1 - \epsilon^{-2.5T}).$$

In this formula the arbitrary constant is not related in any way to initial compression of air voids in unsaturated materials.

kg./sq.cm. increment under consideration. Two alternatives are available for avoiding this impasse. One is to work with linear plots (Figure 26) shaping the synthetic curve in such manner that its slope is everywhere proportional to that of the observed void ratio-compressive strength curve. The second method is to accept the hypothesis that effective intergranular stresses consist of two parts; one of which is dependent on the past consolidation or compaction history of the material; the other being a function of the applied external load and its duration in time. This hypothesis is wholly equivalent to the concept of cohesion. It is readily determined by computation that addition of an intrinsic effective stress of 0.97 kg./sq.cm. to the applied stress increment of 2.81 kg./sq.cm. yields a synthetic consolidation curve parallel to the void ratio-compressive strength curve. The computation, for Figure 27, is as follows:

$$\frac{2.81 + \sigma_0}{0 + \sigma_0} = \frac{11.20}{2.88} = 3.89$$

$$\sigma_0 = \frac{2.81}{3.89 - 1} = 0.97.$$

The values 11.20 and 2.88 are taken by extrapolation from the void ratio-shear strength relationship, Curve I, Figure 27. The synthetic void ratio-effective pressure relationship is shown by Curve II. As previously stated, the method used in constructing Curve II is mathematically equivalent to the a priori assumption that Coulomb's hypothesis is applicable. For any void ratio, Curve II defines $(\sigma_0 + \sigma_{III})$, the effective minor stress, and Curve I defines $\frac{(\sigma_1 - \sigma_3)}{2}$, the maximum shear stress at failure. In these expressions, σ_1 and σ_3 are the total externally applied stresses and σ_{III} represents that part of σ_3 which has become effective through consolidation. Assuming no change in effective stress during the quick shear test, Figure 27 indicates a constant ratio between the effective minor stress and the maximum shear stress:

$$\frac{\sigma_0 + \sigma_{III}}{\frac{\sigma_1 - \sigma_3}{2}} = 0.675.$$

This ratio which does not vary with void ratio determines ϕ as follow:

$$\sin \phi = \frac{\frac{(\sigma_1 - \sigma_3)}{2}}{\frac{(\sigma_1 - \sigma_3)}{2} + (\sigma_0 + \sigma_{III})}$$

$$= \frac{1}{1 + 2\frac{(\sigma_0 + \sigma_{III})}{(\sigma_1 - \sigma_3)}} = 0.596$$

$$\tan \phi = 0.74.$$

The numerical value of cohesion is determined as follows:

$$c = \sigma_0 \tan \phi = 0.72 \text{ kg./sq.cm.}$$

Practically identical results are obtained if the synthetic pressure-void ratio relationship is determined by the first method mentioned above. If that procedure is used, $\underline{c}$ and tan ϕ may be obtained by construction of Mohr's diagram (Figure 26) without pre-determination of σ_0, the hypothetical built-in stress.

The purpose of this discussion has not been to evaluate $\underline{c}$ and ϕ numerically, but to illustrate what is considered to be the physical significance of the relationship between void ratio and shearing strength. It is believed that void ratio is simply an index to the effective intergranular stresses in the soil structure at any time. For the materials and test methods of Table 2, it appears possible that the void ratio is primarily an index to compressive stresses, and that the shearing stresses were of such short duration as to have little effect on the void ratio, or on the normal stresses acting on the planes of maximum shear. It further appears that the time rate of change of void ratio and effective stress is consistant with the principles of Terzaghi's theory of consolidation, if not with its quantitative details. Considering the fact that consolidation theory justifiably introduces several approximations for the purpose of simplifying its mathematical aspects, one should not expect quantitative agreement.

Thus, Mr. Leonards' unique relationship between void ratio and shearing strength is a logical consequence of retention of Coulomb's hypothesis, modified to take into account effective rather than applied stresses. The same opinion is held with respect to Rutledge's correlation between moisture content and compressive strength for saturated clays. There is certainly nothing original or unique in these opinions, which merely reflect two of the classic ideas of soil mechanics.

Returning to the numerical aspects of the writers' interpretation of the tests, it is admitted that numerical precision is lacking. Only in Series No. 4 of Table 2 are there sufficient differences in the times of consolidation to permit an approximate evaluation of intergranular pressures at the end of the partial consolidation period. Most of the Fort Union tests are considered to be close approximations of "Q" tests, notwithstanding their "Qc" label. By coincidence, this series is also one in which the shearing of the specimens caused little or no change in void ratio. In this connection, it is noted that the author's "void ratio at failure" is actually the void ratio after failure and removal of all externally applied loads. In the case of the Series 4 Fort Union material, it appears probable that actual void ratio in the failure zone, during shear failure, was not materially different from the void ratio after partial consolidation and also after failure and unloading. This probability suggests that void ratio in this particular series of tests may be regarded as a sort of "strain-gage" for measurement of compressive stresses during shear. For other materials, and for other rates of application of the shearing stresses, it is very doubtful that such a simple assumption is valid. For example, one would expect the void ratio of Mississippi Loess specimens of Series No. 1 and No. 2, Table 3, to respond fairly rapidly to any change in stress conditions, owing to their relatively high air-void ratios. As another example, one might expect the void ratio of the Fort Union Series 4 material to be responsive to loads

applied during shear testing, if the rate of application were sufficiently slow. Thus in the general case, the relationship between compressive stress, shearing stress, void ratio, and time may not be fully represented by the simple diagram of Figure 27.

It is believed that Mr. Leonards' outstanding development in laboratory techniques points the way toward successful investigation of these relationships. For example, one might begin with a series of initially identical specimens, carefully prepared by the author's techniques, and slowly trace out the void ratio changes due to triaxial compression, experimentally determining these changes in terms of applied stress, time, and effective stress as derived from consolidation theory. Following this calibration of void ratio as a "strain-gage" for measurement of pure compressive stresses, a second series of tests could be performed into which shearing stresses would be introduced, while retaining the same mean applied compressive stresses in successive comparative series. As a more specific example, it is suggested that a unit group of tests on a material such as the Series A, Fort Union might consist of the following:

Applied Total Stresses, kg./sq.cm.				Time in Minutes for Systematic Determination of Void Ratio				
σ_1	$\sigma_2 = \sigma_3$	Mean compressive	Shear					
3.0	3.0	3.0	0.0	0	1440	2880	5760	etc. as
3.4	2.8	3.0	0.3	0	1440	2880	5760	required
3.8	2.6	3.0	0.6	0	1440	2880	5760	to at
4.2	2.4	3.0	0.9	0	1440	2880	5760	least 90%
etc. to shear failure							consolidation	

Although such a group of tests obviously entails an enormous amount of work on a single value of mean compressive stress, it should be worth the effort as a positive indication as to whether shearing strength can rationally be predicted on the basis of Coulomb's and Terzaghi's theories.

Certainly no one will deny that Mr. Leonards' tests establish a relationship between maximum shearing stress and void ratio after failure and unloading of the sample. The categorial statement in the synopsis that "the critical factor that controls the strength of partially saturated, compacted clays is the void ratio at failure" is unacceptable. This statement seems inconsistent with the fact that each series of tests exhibits a different shear strength. The author later seeks to explain this discrepancy by introducing the qualifying condition that this statement is true "for a given set of initial conditions". One could quite logically consider the initial conditions for a shear test to be those existing just prior to the application of the shearing stresses. Also, one could quite properly consider initial conditions to be those existing prior to the "compaction" of the specimens in the large consolidation device. Under either of the alternate definitions of initial conditions, the unique "control" of shear strength by void ratio becomes invalid.

These inconsistencies disappear if one abandons the idea of void ratio as a controlling factor, and restores void ratio to its proper place as a result, not a cause, of stresses. Figure 28 illustrates this viewpoint. It is a schematic pressure-void ratio curve for two materials which were identical in composition and moisture content in the loose state, prior to compaction by static load. Pressures are partial effective intergranular stresses, as distinguished from applied pressures. The term "partial" here excludes any hypothetical effective stresses locked into the material by prior load applications. Specimen A is subjected to an unknown consolidation for an unknown period of time, resulting in an effective pressure and void ratio indicated in the figure by the point (p_a, e_{a1}). The load is removed, and the material rebounds to zero applied load and void ratio e_{a2}. Specimen B is subjected to similar treatment, except that the intensity and duration of the static compaction load results in a lower void ratio after rebound, e_{b2}, as indicated on the figure.

Next, the two specimens are consolidated by an appropriate combination of load intensity and duration to an identical void ratio, e_{ab}, on the figure. Obviously, the effective applied stress necessary to achieve this common void ratio is ordinarily greater for Specimen A than for Specimen B. Consequently, if the conception of friction is retained, the frictional component of the strength of Specimen A would be expected to be greater than for Specimen B. Likewise, recalling that the frictional component with respect to effective intergranular stresses tends to be much greater than the "cohesion" component, one would also expect the total strength of Specimen A to be greater than that of Specimen B at the same void ratio. Finally, the effective stresses on the two specimens would approach equality as the selected common void ratio is lowered, and the two recompression curves approach the common virgin consolidation curve.

Referring to Figures 15 and 20, it is seen that the tendency toward convergence of the strength-void ratio curves is quite apparent as the void ratio decreases, and the effective confinement of the test specimen increases. From the viewpoint presented in this discussion, this comparison of strength at a common void ratio is in itself a refutation of the idea that void ratio, as such, exerts a control over shearing strength.

Although intensely interesting from a research viewpoint, the difference in strength due to minor differences in compaction appears to be of little practical consequence in the field. By way of illustration, assume that Fort Union clay is compacted to the initial conditions of Series 2 and Series 4 in adjacent lanes of an embankment. Further assume that as the embankment is raised, the two lanes are subjected simultaneously to identical applied loads, which impose shearing and compressive stresses in the field in the same manner as they were applied in the laboratory in those tests in which the confining pressure was 2.81 kg./sq.cm. To the extent that laboratory tests are applicable to field conditions, Figure 29 shows what may be expected. The potential shearing strength increases with time as the two materials consolidate and effective stresses are increased. At any chosen time when the consolidation process is interrupted by shearing failure, the strength is practic-

ally identical for the two materials. If, during construction of the hypothetical embankment, the contractor were paid additional money to obtain a dry density of 99 lbs./cu.ft. instead of 97, it is agreed that we have a case of economic overcompaction. If the two densities were obtained at the same price, there seems to be no cause for alarm.

To summarize, profound admiration must be paid to the precise laboratory methods used in the investigation. At the same time, it is necessary to express disagreement with practically all of the conclusions and implications drawn by Mr. Leonards. It is believed that the correlation of void ratio, moisture content, or other like variables has been overemphasized in this paper and by other investigators.

The maximum shearing stress of a soil can be very closely approximated by the expression

$$s = c + n \tan \phi \qquad (1)$$

in which c is a function of the entire past stress-strain-time history of the material, n is a function of time and the total stresses externally applied to the material, and tan ϕ is a variable with such a slight range of values that it may be considered a constant. It is also believed that the void ratio may be defined approximately by

$$e = F_1(\sigma_1, \sigma_2, \sigma_3, t) \qquad (2)$$

with some reservations in the case of clays of the montmorillonite type. This latter expression is nothing more than the assertion that void ratio is the result of all past stress conditions which have existed prior to a chosen time. If the chosen time coincides with failure in shear, there is clearly a relationship between void ratio at failure and shearing stress at failure. It so happens that the relationship

$$s_{max} = F_2(e_f) \qquad (3)$$

can readily be established experimentally under certain conditions. But the relationship implied by equations (1) and (2) are extremely difficult to establish experimentally, even though the theories of Coulomb and Terzaghi point toward fruitful methods of investigation. In the majority of cases, as the author seems to agree, the laboratory solution of equation (3) although undoubtedly true, is of no practical value for the reason that the important variables σ_1, σ_2, σ_3, and t are eliminated entirely. These are precisely the variables which are common to equations (1) and (2), and which in some form must be dealt with in solution of most problems. The true worth of experimental evaluation of equation (3) lies in its apparent utility as a tool in further study of the difficult problems inherent in equations (1) and (2).

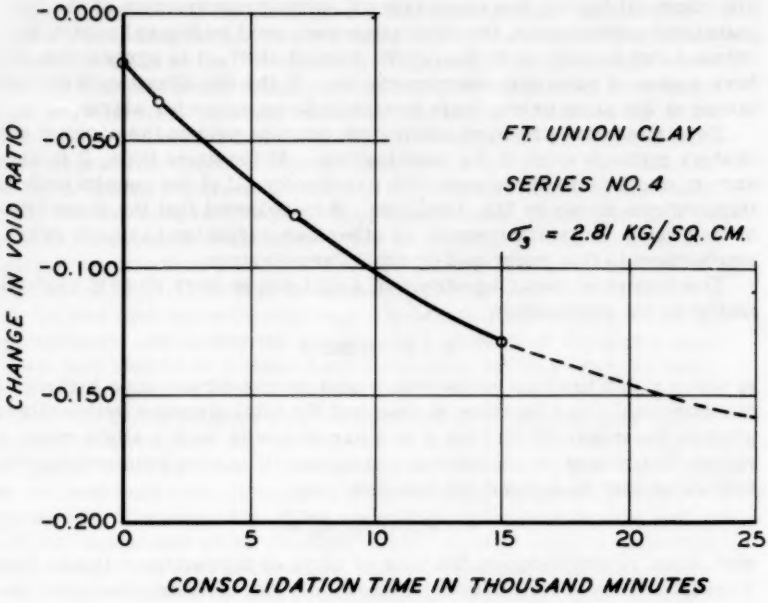
-0.000
-0.050
-0.100
-0.150
-0.200
CHANGE IN VOID RATIO
FT. UNION CLAY
SERIES NO. 4
σ_3 = 2.81 KG./SQ. CM.
0
5
10
15
20
25
CONSOLIDATION TIME IN THOUSAND MINUTES

Fig. 25

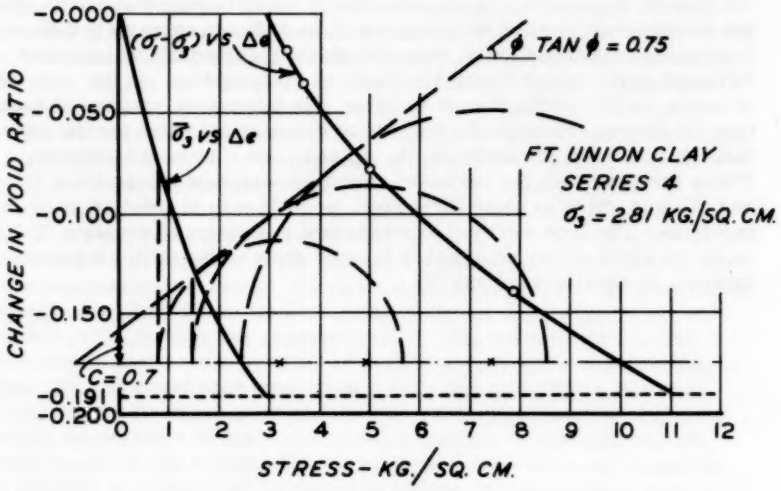
-0.000
-0.050
-0.100
-0.150
-0.191
-0.200
CHANGE IN VOID RATIO
$(\sigma_1-\sigma_3)$ vs Δe
σ_3 vs Δe
ϕ TAN ϕ = 0.75
FT. UNION CLAY
SERIES 4
σ_3 = 2.81 KG./SQ. CM.
C = 0.7
0
1
2
3
4
5
6
7
8
9
10
11
12
STRESS - KG./SQ. CM.

Fig. 26

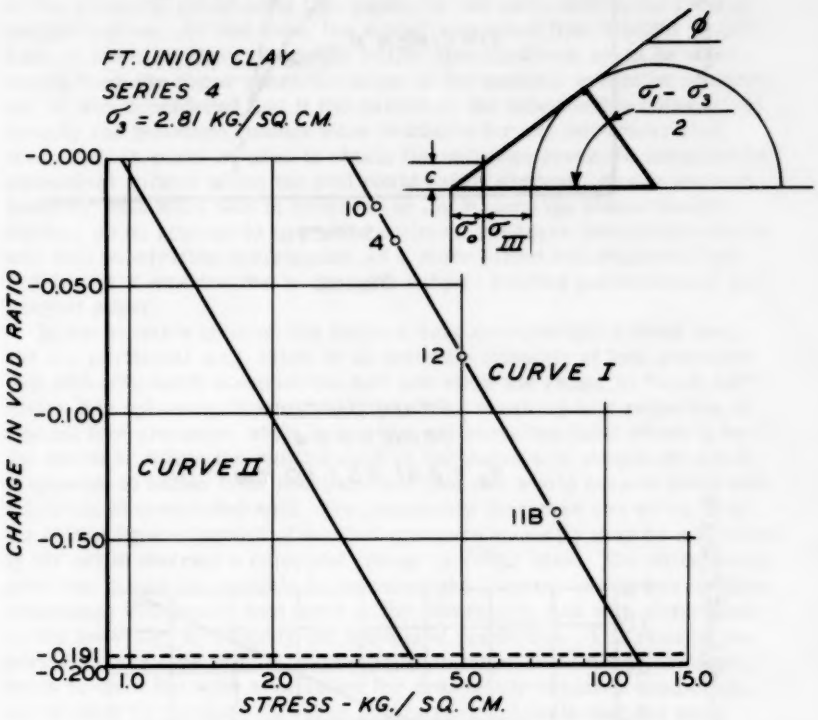
FT. UNION CLAY
SERIES 4
σ_3 = 2.81 KG/SQ. CM.
ϕ
$\frac{\sigma_1 - \sigma_3}{2}$
c
σ_o
σ_{III}
-0.000
-0.050
-0.100
-0.150
-0.191
-0.200
CHANGE IN VOID RATIO
10
4
12
11B
CURVE I
CURVE II
1.0
2.0
5.0
10.0
15.0
STRESS - KG./ SQ. CM.

Fig. 27

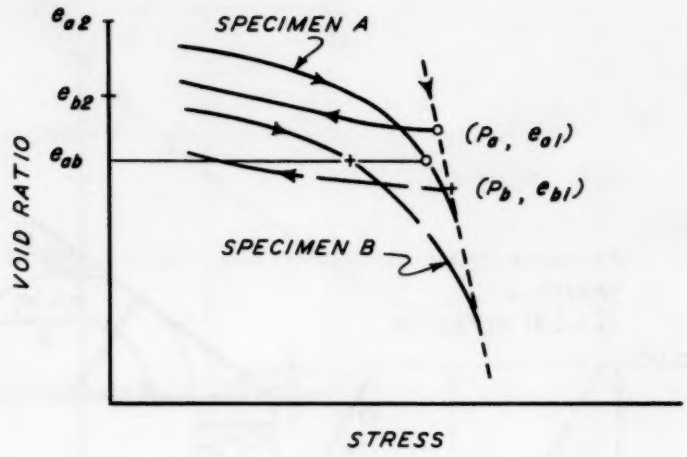
e_{a2}
e_{b2}
e_{ab}
SPECIMEN A
(P_a, e_{a1})
(P_b, e_{b1})
SPECIMEN B
VOID RATIO
STRESS

Fig. 28

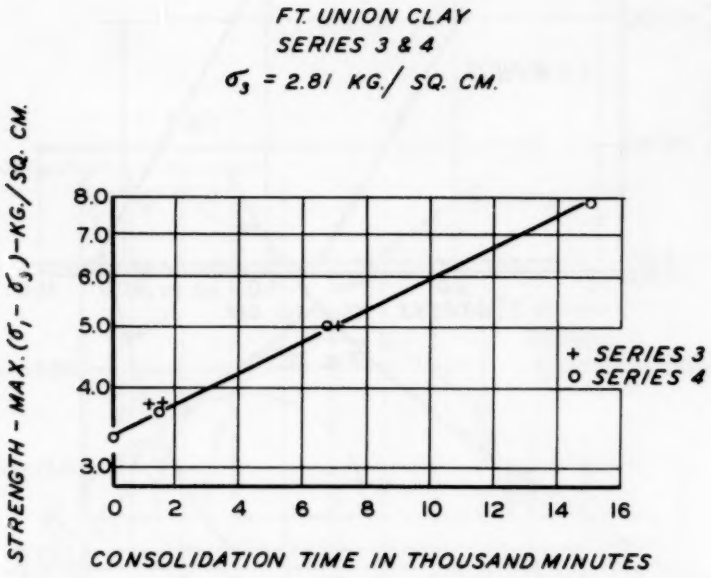
FT. UNION CLAY
SERIES 3 & 4
σ_3 = 2.81 KG./ SQ. CM.
STRENGTH - MAX. $(\sigma_1 - \sigma_3)$ - KG./SQ. CM.
8.0
7.0
6.0
5.0
4.0
3.0
0
2
4
6
8
10
12
14
16
+ SERIES 3
o SERIES 4
CONSOLIDATION TIME IN THOUSAND MINUTES

Fig. 29

DISCUSSION OF FIELD PENETRATION TESTS FOR SELECTION OF SHEEPSFOOT ROLLERS

B. A. KANTEY, A.M. ASCE.[1]—Since 1951, when the writer first heard of the research reported in this paper, he has been looking forward to its publication. At that time, the writer suggested that it might be possible to evolve a theory, whereby roller specifications could be determined from the shear characteristics of the specific soil to be compacted. It was considered that if the curves of the relationship between c, ϕ, density and moisture content were available for any particular soil, these curves might be used to obtain the optimum pressure intensity for sheepsfoot rollers which the soil could safely support. It was learned, however, that work was in progress at the Waterways Experimental Station, on an attempt to correlate optimum pressure intensities of rollers with penetration resistances, as a more direct and empirical approach, and it was decided to drop the subject pending publication of the present paper.

In the writer's opinion, the authors have convincingly proved that, for any particular soil, there is an optimum intensity of foot pressure that will efficiently compact the soil and allow the roller to "walk out". Above this optimum, the soil fails by shear resulting in a reduction of applied foot pressure, while below this optimum, wasteful effort is being exerted. While the method used by the authors is simplicity itself, it appears to suffer from the drawback that one would have to start with a section of compacted soil. The possibility therefore can arise, that the initial "guesstimate" of the foot pressure intensity may be sufficiently far out to warrant a complete change in roller size. The authors suggest that it may be possible to correlate the Proctor needle penetration resistance with model foot tests in the laboratory, and with sheepsfoot roller behavior, to simplify the method of prediction. The results reported to date (see Fig. 7), particularly in the soils of lower strength, seem to show too wide a variation for reasonably accurate prediction, and it must be realised that it is in this range of soils that the most trouble can be expected with lack of "walking out".

It is possible that this variation may be reduced by the introduction of a size factor, such as Housel's perimeter area ratio, and the writer would appreciate the authors' views on this aspect. For earth dam work, however, it is often very necessary to carry out triaxial tests on samples of laboratory compacted material, and, with such results available, the writer wonders whether these could not be used, along the lines suggested above, to arrive at a more accurate initial estimate for the required roller foot pressure intensities.

1. Cons. Engr. Kantey & von Geusau, Cape Town, South Africa.

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a. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
b. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.
c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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